1 INTRODUCTION

The present paper relates to the design of the underwater sand embankment on soft soil (dredged material). The water depth to the sediments level is about 20 m. As detailed in the previous chapter, the soil profile at the site consists of an upper soft layer overlying sand and a deep tertiary clay layer.

The characteristics of the problem and the conditions of the foundation soil required improvement of the foundation (by deep mixing techniques) soil on the one hand and construction in stages on the other hand in order to prevent early instability. Moreover, restrictions imposed on the total time of construction led to adopt extra reinforcement elements such as geotextiles in the embankment body (Figure 1).

On the basis of the previous design alternatives, a more refined design has been proposed (Figure 2). A partial level of improvement was performed here at the toes of the embankment.

The research is focusing first on a better understanding of the soft soil particle-cement interacting, implementing seven different type of cement and various mixing techniques. The second issue relates to the calibrating and adjusting the large strain consolidation theories through monitoring on site, of the soft bearing layer.

The general profile of the embankment and soil conditions are illustrated in Figure 2. The soil profile consists of about 8m of soft material, 2m of sand and a deep highly overconsolidated clay (Boom clay). The water depth to the sediments level is about 20m. The soft material was once subjected to vacuum consolidation to accelerate the self-weight consolidation. That means that there exist drains installed in the soft material accelerating the pore water pressure dissipation.

Geotextile reinforcement was horizontally installed and distributed every 2m on the upstream face slope also as a protective measure against wave action, to reduce the waiting time between subsequent construction phases and to provide extra safety.
2 DEEP MIXING APPLICATION

2.1

Deep mixing can be executed in mass or column stabilization and both can be applied in many different ways. The soil can be stabilized either by forming columns of stabilized soil (so-called column stabilization) or by stabilizing the entire soil volume (so-called mass stabilization). Figure 3 gives some examples of the configuration of columns and Figure 4 suggests the application for the combined mass and column stabilization.

Globally, local technologies are developed for new applications and for specific geographic areas often by innovative contractors who are seeking to develop their own variant of the method to match a particular project challenge.

The large number of existing DM techniques have been classified by Bruce (2001) on the following simple basis:

- Is the cementious material injected wet (W) or dry (D)?
- Is the binder mixed with the soil by means of rotary energy (R) only or is the mixing enhanced by high-pressure jet (J) methods?
- Is the mixing action only occurring near to the drilling tool (E), or is it continued along the shaft (S) for a significant distance above it, by way of augers and/or paddles?

The stabilizing agents are in most of the cases, Portland cement and lime, but also other binders have been more recently implemented. Some of these new binders have been designed for clayey soils with high natural water contents or organic soils for which ordinary Portland cement or lime is not very effective. A brief description of the mechanism of stabilization of lime and cement is given in the following sections.

2.2

When mixed with clayey soil, quicklime (CaO) reacts with pore water of the soil to become slaked lime Ca(OH)₂. This reaction takes places quickly and releases a large amount of heat. This brings in a reduction of the natural water content of the soil which already represent an improvement on its shear strength.

In presence of sufficient water the hydrated lime dissolves into Ca²⁺ and OH⁻. Then, Ca²⁺ ions exchange with cations on the surface of the clay minerals. The cation exchange reaction alters the charac-
teristics of the water films on the clay minerals. In general the plastic limit of the soil is increased, reducing the plasticity index. Furthermore, under a high concentration of hydroxyl ions (high pH), silica and/or aluminum in the clay minerals dissolve into the pore water and react with calcium to form a water insoluble gel of calcium-silicate or calcium-aluminate. This pozzolanic reaction goes on as long as the high pH condition is maintained and calcium exists in excess. Figure 5 illustrates the lime stabilization mechanism in which the product of the pozzolanic reaction cements the clay particles together.

Figure 5. Lime stabilization mechanism (CDIT, 2002)

2.3

The most commonly used cement types for stabilization are Portland cement and Blast furnace cement. Portland cements are inorganic binders obtained by grinding to a high fineness, Portland clinker alone, or most commonly in combination with calcium sul fate (gypsum) acting as a set regulator.

In ordinary Portland clinker, tricalcium silicate (C3S) is the most abundant phase present in amounts between about 50% and 70%. Dicalcium silicate (C2S) usually constitutes 15-30% of the clinker. Typical amounts of tricalcium aluminate (C3A) are 5-10% and of the ferrite phase (C4AF) 5-15%. During the hydration of the cement a C-S-H phase is formed and Ca(OH)2 is released. The first hydration product has high strength which increases as it ages, while Ca(OH)2 contributes to the pozzolanic reaction as in the case of lime stabilisation.

Figure 6 illustrates the cement stabilization mechanism. Immediately after mixing it is possible to identify clay clusters and cement paste as separate phases. Next, the strength of the stabilized soil will gradually increase due to pozzolanic reactions within the clay clusters and hardening of the cement paste.

Blast furnace cement is a mix of Portland cement and blast furnace slag and shows a similar stabilization mechanism. Finely powdered slag does not react with water but it has the potential to produce pozzolanic reaction products under high alkaline conditions. The SiO2 and Al2O3 contained in the slag are actively released by the stimulus of the large quantities of Ca2+ and SO42- from the cement, so that a hydration product is formed for which the long-term strength is enhanced. The complicated mechanism of stabilization has been simplified by Saitoh et al. (1980) in Figure 7 for the chemical reactions between clay, pore water, cement and slag.

Figure 6. Cement stabilization mechanism (CDIT, 2002)

Figure 7. Chemical reaction among clay, cement, slag and water (Saitoh et al., 1985)

2.4

The installation of columns stabilized with deep mixing methods requires the use of specially designed machinery which basically consists of:

• A binder feeding unit
• A soil mixing machine for injection of binder into the ground

Figure 8 illustrates a scheme of a typical DM column installation machinery. The binder feeding unit comprises various devices that measure the quantity of the ingredients in the admixture and transport it to the soil mixing machine. The plant generally include
A variety of soil mixing tools have been manufactured for various improvement purposes, ground conditions and special applications. In general, two categories of mixing tools can be identified:

Blade-based mixing tools: These tools have wide blades for excavation, in addition to paddles and/or short blades for cutting and mixing. The mixing process is mainly carried out at the tip (or close to the tip) of the tool (e.g. Figure 9). The blades have a variety of shapes, dimensions and orientations.

Auger-based mixing tools: These tools have discontinuous or continuous helical augers for drilling, in addition to paddles and/or short blades for cutting and mixing. The mixing process is carried out in portions along the drilling shaft.

3 DEEP MIXING ASSESSMENT ON AN UNDERWATER SITE

3.1

The soil investigated is a soft deposit of fine grained material, result of a prolonged sedimentation and self-weight consolidation process of dregs removed from the waterways within the harbor of Antwerp. In many harbor areas all around the world, there is an increasing need of reclaimed land. This fact has encouraged the design and ongoing construction in the Antwerp harbor of a partially submerged 27-m high sand embankment on the soft material previously mentioned. Obviously, the presence of such soft foundation layer caused concern for the overall stability; therefore partial improvement of the material by deep mixing was proposed.

This section focuses on the laboratory and field investigation carried out for the evaluation of the improvement of the soft fine grained soil with cement. Initially, the effect of several types of cement was studied in the laboratory. From those results a blast furnace cement was chosen as most suitable for the application in the field. Finally, a field inspection was carried out to assess the actual improvement of the deep mixing columns installed by the SSI technique (SSI is a technique patented by HSS, DEME-Dredging International-Belgium).

3.2

The soil collected from the soft deposit was thoroughly homogenised and remolded prior to mixing with cementing agents.

A dough mixer was employed here to mix the soil and a slurry of cement. The dosage of binder for mixing with soil was set to 275 kg/m³, the water/cement ratio of the slurry was set to 0.8 and a mixing time of about 10 minutes was implemented. This extended mixing time was meant to allow for more intensive mixing; however, only a slight difference in strength was observed when compared to specimens mixed for 5 minutes (less than 5% after 7 days).

Cylindrical specimen with a diameter of 57 mm and a height of 115 mm were prepared by pouring the mix into split plastic moulds. The moulds were later sealed with paraffin film and stored under water in a conditioned room at 10°C with no overburden whatsoever acting on the specimen. In addition, some specimen were cured under water at 20°C in order to study the effect of the temperature on the development of the improvement.

At the initial stage of this project, a number of different types of cement have been employed in the laboratory. A short description (according to EN 197-1) of the binders is given below:

- Binder A, B, and C are all blast furnace cements, CEM III. Binder C has the greatest blast furnace slag content (CEM III/B). Binder B and C classify at a nominal strength of 42.5 MPa while binder A has only 32.5 MPa.
- Binder D is a Portland cement, CEM I, with a nominal strength of 52.5 MPa.
• Binder E is a commercially available binder specifically designed for stabilization of soil.
• Binder F is a cement typically used for soil grouting purposes.

A large number of unconfined compression tests have been performed at several time intervals (i.e. 7, 14, 28, 56, 84, 120, 240 and 550 days). The results of the testing programme on specimen cured under water at 10°C have already been reported by Van Impe et al. (2004b) and are summarized in Figure 10.

From the group of binders tested here, it seems that the blast furnace cements (binders A, B and C) perform quite well, showing a continuous increase of the UC strength. Binders B and C (both CEM III 42.5) do show an unconfined compressive strength of the order of UCS ≈ 2.2 MPa after 550 days. The Portland cement (binder D), on the other hand, allows for more rapid hardening in the first days. In fact, it shows the highest UC strength during the first month. However, the improvement provided by Portland cement seems to decline afterwards for some period to finally pick up again after some 3 months. The understanding of why systematically this “interval” of the interplay cement-soil occurs is subject to further research today. Anyhow, the final compressive strength of Portland cement remains lower than that given by the blast furnace cements B and C. The other binders (E and F) seem to produce little improvement for such high dosage (UCS ≈ 0.7 MPa after 550 days).

Figure 10. UCS of cement stabilised specimen cured under water at 10°C

The strain at failure of specimen cured under water at 10°C, illustrated in Figure 11, was measured externally (from top to bottom cap of a triaxial cell) by LVDT. The figure provides some information about the ductility of the stabilised mass. In spite of some scatter it seems possible to establish a general tendency of behaviour for each binder mix.

Figure 11. Strain at failure of cement stabilized specimen cured under water at 10°C

Overall, the strain at failure (ranging from 0.9% to 4%) decrease rapidly with increasing UC strength. The brittleness increases obviously with increasing UCS values. From the results it can also be deduced that, as ageing increases, specimen mixed with Portland cement tend to yield at smaller axial strains than specimen mixed with blast furnace cement, even tough the strength of specimen mixed with Portland cement is considerably lower.

Measurements of small-strain modulus were also performed by means of bender element testing at different time intervals for some specimen (cured under water at 10°C) mixed with blast furnace cement (Binder C) and with Portland cement (Binder D) only.

The bender element test set up employed here is given in Figure 12. The principle of this non-destructive method is simple and well know from literature (Dyvik & Madshus, 1985). As an example, Figure 13 illustrates the S-wave arrival time measured for specimen stabilised with blast furnace cement at several curing time intervals using an input sinusoidal pulse with a frequency of 4 kHz. Each specimen was tested for unconfined compression to measure UCS. As expected, a rather linear relationship between $G_0$, $E_0$ and UCS is observed.

Figure 14 summarizes the Young’s modulus at small strain $E_0$ evaluated here for specimen mixed with Portland and blast furnace cement. The modulus for the Portland cement was found to be slightly higher but still, a single linear correlation has been proposed for both cements: $E_0 \approx \frac{714}{(UCS)}$.

Similarly, Figure 14 illustrates the secant Young’s modulus evaluated from unconfined compression tests. Even if trend shows some scatter, the data could be more or less linearly correlated to UCS as well. It has been estimated as $E_{50} \approx 110$ (UCS). This trend is considerably low when compared to the
Japanese experiences reported by Saitoh et al. (1980) where $350 \text{ (UCS)} < E_{50} < 1000 \text{ (UCS)}$; however, it falls within the range of many other correlations proposed worldwide in the literature (Porbaha et al., 2000). Overall, the modulus of the Portland cement is slightly higher than that given by the blast furnace cement. In general, $E_0$ remains about 7 times $E_{50}$.

In an attempt to more reliably recreate the conditions in the field, a large cylindrical specimen with a height $H \approx 0.8$ m and diameter $\varnothing \approx 0.6$ m was prepared in the laboratory employing blast furnace cement, with the aim of evaluating and monitoring the temperature changes due to exothermic reactions within the stabilized mass.

The virgin soil was kept at a temperature of $10^\circ$ C prior to mixing. After mixing of the soil and blast furnace cement slurry in a concrete mixer, the stabilized mass was poured into a large plastic mold (also stored at $10^\circ$ C and with the above mentioned dimensions) where eight temperature transducers (labeled T1, T2...T8) were installed at different locations within the sample. A few small cylindrical specimen were also prepared and cured (under water at $10^\circ$ C) following the ordinary procedure described in a previous subsection.

The temperature measurements within the stabilized mass over a period of 56 days are illustrated in Figure 15. The readings of all temperature transducers do show a common trend. Immediately after mixing a sudden temperature increase was observed. After 3 days a maximum temperature of about $25^\circ$ C was reached. Finally, the temperature in the large specimen seems to gradually decrease; after 56 days, the temperature (about $11.7^\circ$ C) leveled out at values only slightly over the conditioned room temperature ($10^\circ$ C).

By the end of the temperature monitoring some core samples were taken from the large specimen. Figure 16 shows the UCS of such core samples.
Figure 16. UCS of core specimen from a large stabilized specimen

The figure also indicates the UCS of small specimen from the routine laboratory testing as described in a previous subsection. Clearly, the UCS of the large specimen cores doubles the UCS values of the small specimen. This suggested that the transient temperature increase due to the exothermic reactions within the large specimen were imposing such notable difference. Indeed, the larger the sample, the slower the heat dissipation and so the higher the UCS to be expected.

In order to study the effects of the curing temperature on the UC strength of the stabilized dredged material an extra series of tests has been carried out; this time on small specimen mixed with blast furnace cement, cured under water at 20° C. The results (Figure 17) demonstrated that the strength of the samples stabilized with blast furnace cement is notoriously affected by the temperature. The hydration of the blast furnace cement clearly benefits from high temperatures; in fact, the UCS of samples cured under water (up to 200 days) at 20° C is, at all times, about 1.7 to 2 times larger than the UCS of specimen cured at 10° C. The experimentation for the evaluation of properties of the cemented soil in the field consisted of core sampling of specimen from trial columns to proceed later on with unconfined compression tests in the laboratory.

The trial deep mixing columns (Ø ≈ 1.9 m) were installed in the site (underwater) with the SSI technique from a jack-up platform. Only blast furnace cement (Binder C) was used for the field experimentation. The cement was mixed with water an transformed into a slurry (w/c =0.8) on land. The cement slurry was pumped to the jack-up platform by means of floating pipes. In order to optimize the column installation rates the jack-up platform was provided with a moon pool to allow the installation of 22 to 24 columns in each zone covered by the platform. State of the art positioning systems ensured a very precise location of each column.

The SSI technique makes use of pressurized mixing by means of a mixing tool provided with 2 sets of nozzles distributed all along the full diameter of the column (Figure 18) The mixing tool is fixed to a main drilling rod and each set of nozzles is connected to independent injection systems (Figure 19).

A high-pressure injection system (of the order of 20 to 30 MPa) cuts the soil and allows for intense mixing while the low-pressure injection system (up to 5 MPa) adds the remaining amount of cement slurry to fulfill the required dosage. Injection of the cement slurry takes place during the downwards and upwards operation of the drilling rod. A more detailed description of the installation and performance of the method can be found elsewhere (Van Mieghem et al., 2004). All drilling and injection execution parameters were automatically controlled to accomplish a binder dosage of 275 kg/m³ approximately. A number of core specimen (Ø = 100 mm) obtained over the full length of the trial deep mixing columns were tested to evaluate the actual unconfined compressive strength.
Figure 19. SSI set up and methodology

Figure 20 compares the strength evaluated from core specimen to that obtained from laboratory prepared specimen (after a curing period of 56 days). The UCS in the field ranges from 2 to 5 MPa in the upper 5 m and from 5 to 8 MPa in the lower zone, where a higher content of sand was observed. While, starting from the laboratory investigation, a quite optimistic UCS value remains below 0.9 MPa for similar conditions.

4 LABORATORY VERSUS IN-SITU BEHAVIOR

4.1 In order to evaluate the reasons for discrepancy between laboratory and field test results, a laboratory reconstituted and mechanically mixed specimen on the one hand and undisturbed core samples from columns on site, on the other hand, were analyzed by means of Scanning Electron Microscopy (SEM), aiming at investigating the microstructure and composition of each specimen.

Figure 21 shows both specimen (4cm x 4cm x 1cm) carefully cut with a water-cooled sawing system starting from stabilized samples, either mixed in the laboratory, or in the field by the SSI technique. It is obvious, already from this pictures, that the specimen differ in texture. Until the moment of the microscopic analysis, the sample from the laboratory was about 300 days old and had been kept sealed, under water, in a T=10 C conditioned room. On the other hand the sample from the field was approximately 270 days old; this specimen was cored from a trial SSI column about 3 months after its installation and then kept under water as well, until the day of SEM analysis.
The presence of large pores in a considerable amount in the laboratory specimen is evident. This is by far less pronounced in the SSI improved field specimen where a more compact and more homogeneous texture can be observed. At this point it may be stated that the mixing in the laboratory (by means of a dough mixer) could have caused the incorporation of air bubbles (large pores). Pores of smaller diameter observed in both specimen probably are produced during the cement hydration process. Figure 22 does illustrate the same specimen but, this time, with an amplification factor of 1200. It is again quite clear that the microstructure is diverse.

The specimen from the field has in general a much more homogeneous structure with a more regular distribution of hydration products, such as the calcium silicate hydrate (C-S-H phase) and the calcium hydroxide (CH). On the other hand, the mechanically mixed laboratory sample shows a rather heterogeneous skeleton where the unaged morphology of the C-S-H phase suggests still a lower degree of hydration.

Here it may also be suggested that the much more intensive high pressure mixing in the field did play an important role (the specific area around each soil particle could be reached by the binder, by far better). It seems that the high-pressure SSI mixing in the field has improved the distribution of cement around the soil particles and as a consequence a faster hydration and hardening has been taking place. In the laboratory, where purely simple mechanical mixing with a dough mixer was put into practice, the cement may have not been so well distributed and is only reaching clusters of soil particles.

Finally, Figure 23 illustrates the samples from the laboratory and the field with an amplification factor of 1700. Also here the same pattern was observed: the structure of the field specimen looks much more homogeneous than the laboratory specimen. A matured C-S-H phase can be recognized in the field specimen together with uniformly distributed CH crystals that cover almost completely the soil.

On the other hand, the mechanically mixed laboratory sample shows a much more heterogeneous composition including also ettringite (AFt phase) that is formed during the early hydration process (this phase is usually absent in matured and well hydrated cement pastes (Odler, 2000)). Overall, judging on the morphology of the different cement hydration products in the pictures, a by far less advanced degree of hydration could be perceived in the laboratory prepared samples.

5 EXCESS PORE WATER PRESSURE

5.1

Figure 24 summarizes the measurements of excess pore water pressure (PWP) in the foundation soil during construction. Significant differences can be observed between measurements within the SSI-improved zone and the non-improved zone.
As expected, the excess PWP in the improved zone is considerably smaller than in the non-improved zone, showing that the SSI columns are indeed taking up an important portion of the load. On the other hand, the excess PWP in the non-improved zone closely follows the stage construction loading history and it is only after about a year of construction that significant consolidation slowly takes place.

At this moment a more extended waiting period is being allowed so that enough strength is gained (due to consolidation) in the foundation soil to continue with the construction activities of the second phase.

Figures 25 and 26 show the measured settlements profiles in the nonimproved and the SSI improved foundation zones respectively. As expected, the largest settlements are observed in the non-improved area were up to now a maximum settlement of the order of 1.2m was measured. On the other hand, the maximum measured settlement in the SSI-improved zone is of the order of 0.6m.

Figure 25: Settlement profile at the interface between embankment sand and foundation layer within the non-improved zone

Figure 26: Settlement profile at the interface between embankment sand and foundation layer within the SSI improved zone

Figure 27: Settlement at the SSI-improved and non-improved foundation zones
Figure 27 compares the average settlements measured in both zones, SSI-improved and nonimproved. It can be observed that the current settlement in the nonimproved zone is about half the total settlement estimated for the present surcharge.

REFERENCES

P. Van Impe. 1999. Consolidatie van verzadigde, sterk samendrukbare poreuze media. Final degree work, Faculty of applied sciences, University of Ghent